Seismic vulnerability of Modern Architecture building's - Le Corbusier style: A case study

H. RODRIGUES[1], J. FONSECA[2], A. COSTA[1], H. VARUM[3] AND A. TOSTÕES[4]
[1]*Universidade do Porto, Faculdade de Engenharia, Porto, Portugal*
[2]*Universidade do Porto, Faculdade de Arquitectura, Porto, Portugal*
[3]*Universidade de Aveiro, Civil Engineering Department, Aveiro, Portugal*
[4]*Universidade Técnica de Lisboa, Instituto Superior Técnico, Lisbon, Portugal*

ABSTRACT

In Portugal, at the end of the World War II, a new generation of architects emerged, influenced by the Modern Movement Architecture, born in Central-Europe in the early twenties but now influenced also by the Modern Brazilian Architecture.

They worked with new typologies, such as multifamily high-rise buildings, and built them in the most important cities of the country, during the fifties, reflecting the principles of the Modernity and with a strong formal conception inspired in the International Style's codes.

Concrete, as material and technology, allowed that those "Unity Centre" buildings become modern objects, expressing the five-point formula that Le Corbusier enounced in 1927 and draw at the "Unité d'Habitation de Marseille", namely: the building lifted in *pilotis*, the free design of the plan, the free design of the façade, the unbroken horizontal window and the roof terrace.

In Lisbon, late forties urban plans transformed and expanded the city, creating modulated buildings repeated in great extensions – that was a progressist idea of standardization. The Infante Santo complex is a successful adaptation to the Lisbon reality of the Modern Urbanism and Architecture.

In the fifties, it was built a large number of Modern housing buildings in Lisbon, with structural characteristics that, in certain conditions, can induce weaknesses in structural behaviour, especially under earthquake loading. For example, the concept of buildings lifted in *pilotis* can strongly facilitate the occurrence of soft-storey mechanisms, which turns these structures very vulnerable to earthquake actions.

The development and calibration of refined numerical tools, as well as, assessment and design codes makes feasible the structural safety assessment of existing buildings. To investigate the vulnerability of this type of construction, one building representative of the Modern Architecture, at the Infante Santo Avenue, was studied. This building was studied with the non-linear dynamic analysis program PORANL, which allows the safety evaluation according to the recently proposed standards.

INTRODUCTION

The 3rd parcel from the Unity Type A in the Infante Santo Avenue, in Lisbon, is a singular example of Modern Housing Project in Portugal [Tostões, 1998]. It's a building inserted in an Urban Plan initialized by Alberto José Pessoa (1919-1985), in 1947, when he joined the "Urbanization Study on the Protection Area of Palácio das Necessidades" in Câmara Municipal of Lisbon (CML). It consisted on dividing and expropriating land parcels for the new Infante Santo Avenue, which completed the ring created by De Gröer's Municipal Plan (1938-1948).

Between 1949 and 1951, CML and A. Pessoa began the urbanization area process with "Beforehand Project for the Housing Project, Market and Shops Complex in Infante Santo Avenue Central Area", which included 31 parcels along the new Avenue. The transverse section of the proposal reveals a volumetric composition that recreates the landscape, showing an obviously modern matrix influence. The buildings look like free objects claiming that "architecture is an awareness game, with correct and magnificent volumes united under the light" [Le Corbusier, 1923], with rectilinear surfaces where we can see "the generators revealing simple forms" [Le Corbusier, 1923] and with a plan order that expresses a primary determinate rhythm "with consequences extended from the simplest to the most complex, respecting the same law. A unity law is the law of a good plan: simple law and infinitely modulated" [Le Corbusier, 1923].

In 1954, A. Pessoa assumes a "Construction Project of the Infante Santo Avenue between the old Aqueduct and Santana à Lapa Street" and he provides, in CML, the 1st Work Team with the architects Hernâni Gandra and João Abel Manta, and Jordão Vieira Dias, the engineer responsible for the structural project.

UNITY TYPE A

This team has done the execution project of the Unity Type A, which are composed by one housing block and another one to the commercial shops. These models are repeated from the 1st to the 5th parcel.

The place of the project is located in the western part of the city, near the Tejo River, between two established urban areas. The housing volumes are displayed parallel to each other and perpendicular to the Avenue. They draw a 45 degrees angle with the river, establishing a strong visual relation with the landscape. This position is ruled by a correct orientation and insulation because the shopping block is opened to the pedestrian way, where public movement is, and the housing block solar exposition is correct. The arrival itinerary is done by the Avenue where can be found the commercial unities, that limit space for the vehicles circulation, and above them appears the suspended 5 blocks. It creates an expressive image of architectural kinetic movement, a conjugation of vertical volumes and a horizontal effect by the repetition series, transmitting a concept so connected to the Modern Movement – the Velocity.

In the block relationship with the ground, there is a clear reference to the 1927 Le Corbusier point o architecture – "the house assents in pilotis"; witch is assumed with big formal value. The block plan is rectangular with 11.10m width and 47.40m length (figure 1). The building has the height of 8 habitation storeys plus the *pilotis* height at the ground floor. The "free plan" is also a reference because the house was conceived in a way of flexibility in use. But, the 12 structural plane frames define the architectural plan of the floor type, with 6 duplex apartments. The distance between frame's axes is 3.80m. Each frame is supported by two columns and has one cantilever beam on each side with 2.80m span, resulting in 13 modules with the rhythm: A-B-B-B-B-B-B-B-B-B-B-B-A. The two A modules are associated with 2 B modules making 2 house types, the other B modules create 4 house types.

The access to the houses consists on an innovation. It as clear functional system, with 3 halls in the ground floor connected to 6 apartments each. This rational distribution access system reveals a public area economy inside the building. The service circulation is the block's "dorsal spine", an exterior gallery with 34m length, semi closed with vertical concrete slabs connecting the house's entrances with the kitchen's doors. This horizontal movement for people and goods is afterwards conducted to the ground floor or to the terrace by a staircase and a functional cargo-elevator.

Fig. 1. General views of the building block under analyses

The structural plan presents an adequate solution to the architecture's objectives. "From the beginning of the studies, it was a permanently concern, to conceive a resistant structure concept, simple, elegant and economic. And it looks like the objective was well succeeded because since the beginning of the project elaboration there wasn't any need for changing the primitive structure".

STRUCTURAL PLAN

The structural plan for the housing block of the Unity Type A is comprised by 12 transversal reinforced concrete frames, formed each one by two columns and 3 beams at each storey, two of them in cantilever.

The structural design was initially made just for vertical loads, without considering the columns bending moment. Afterwards, new designs were developed, now considering in a simplified way the horizontal loads, corresponding to the wind, using the Cross's method in the bending moments distribution calculation. J. V. Dias does not considered in its design the seismic action. He refers "the low probability of simultaneous occurrence of wind and seismic actions in the same direction and at their maximum intensity". Dias concludes that "this building-type has superior safety conditions than the majority of Lisbon's buildings".

Later, the importance of considering seismic actions in structural element's design was recognized. It was delivered a new design project according to an article of Maria Amélia Chaves and Bragão Farinha published in "Técnica" Magazine. Horizontal forces, proportional to the floor's mass were considered in the frame nodes. But, the structural analysis was only made in the transversal direction. The structural engineer concludes that wind forces induce larger demands than seismic loads, resulting in larger cross-sections.

The Engineer Ramos Cruz, responsible for the construction, did a new design project for the 3rd parcel. He presented new calculations based on the primitive project, but he changed the original structural floor by a reinforced concrete slab. He advocates that with this continuous rigid slab, rigid diaphragm behaviour is guaranteed. Another modification is the introduction of reinforced concrete walls in the staircases at the ground floor, one of them was continuous in all binding's height. However, these RC elements were not detected in the technical visit to the building [Miranda et al., 2005].

GENERAL MODELS DESCRIPTION

Nowadays, in the analysis of structures subjected to seismic actions, the use of non-linear behaviour laws and hysteretic rules reveals a great advantage, because it makes possible a more rigorous representation of the seismic structural response.

To simulate the structural behaviour of the building presented in the previous section, it was used a computer program PORANL, that contemplates the non-linear bending behaviour of RC elements (beams and columns) and the influence of the infill masonry panels.

Each RC structural element is modelled by a macro-element defined by the association of three bar finite elements, two with non-linear behaviour at its extremities (plastic hinges), and a central element with linear behaviour, as represented in figure 2.

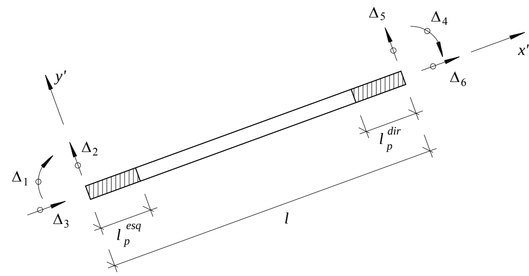


Fig. 2. Frame macro-element [Varum, 1996]

The non-linear monotonic behaviour curve of a cross-section is characterized through a tri-linear moment-curvature relationship, corresponding to the initial non-cracked concrete, concrete cracking and steel reinforcement yielding [Varum, 1996]. The monotonic curve is obtained using a fibre model procedure (see figure 3), from the geometric characteristics of the cross-sections, reinforcement and its location, and material properties.

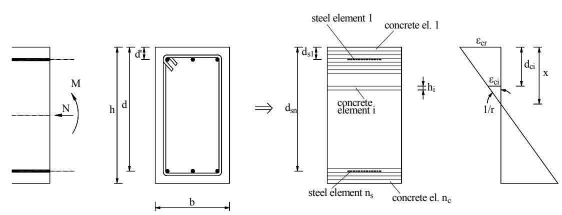


Fig. 3. General fibre model for RC elements

The non-linear behaviour of the plastic hinge elements is controlled through a modified hysteretic procedure, based on the Takeda model, as illustrated in figure 4. This model developed by Costa [1989] represents the response evolution of the global RC section to seismic actions and contemplates mechanical behaviour effects as stiffness and strength degradation, pinching effect, slipping, internal cycles, etc.

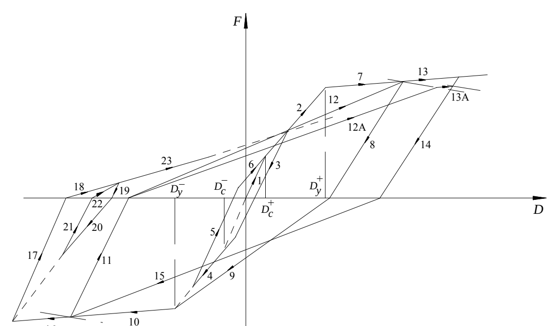


Fig. 4. Hysteretic model for RC elements [Varum, 1996]

To represent each infill masonry panels, an improved macro-model, based on the diagonal equivalent struts model, is used (figure 5). The proposed macro-model was implemented in the non-linear structural analysis program PORANL [Rodrigues et al., 2005]. The macro-model adopted represents the non-linear behaviour of an infill masonry panel and its influence in the global RC structural behaviour under static or dynamic loading.

The monotonic behaviour curve of each panel depends on the panel dimensions, eventual openings dimensions and position, material properties (bricks, mortar, and plaster), quality of the handwork, interface conditions between panel and surrounding RC elements, and can be obtained from empirical expressions or experimental results [Zarnic and Goctic, 1998; Varum, 2003].

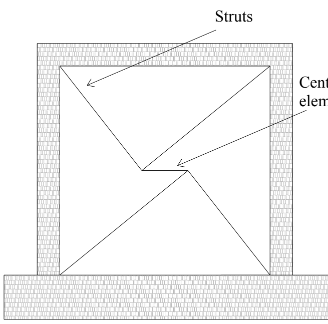


Fig. 5. Infill masonry panel macro-model

The non-linear behaviour of the infill masonry panels subjected to cyclic loads is controlled through an hysteretic procedure and rules, illustrated in figure 6, and represents mechanical effects as stiffness and strength degradation, pinching effect, and internal cycles.

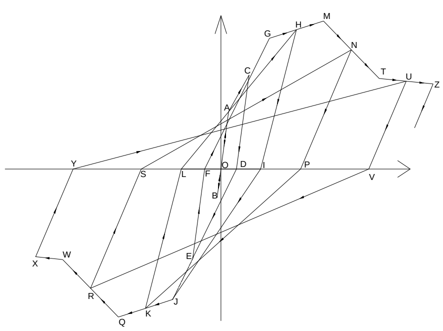


Fig. 6. Hysteretic model for infill masonry panels
[Rodrigues et al., 2005]

DESCRIPTION OF THE STUDIED STRUCTURE

The main objective of the work presented in this paper was to investigate the behaviour of Modern Architecture Lisbon buildings, and their weakness under seismic loading.

The building geometry and dimensions of the RC elements and infill walls were given in the original project [1950-1956], and were confirmed in the technical visits [Miranda et al., 2005]. As already presented in a previous section, the building under study has nine storey and the structure is mainly composed by twelve plane frames oriented in the transversal direction (direction Y, as represented in figure 7). The building was analysed with a simplified plane model for each direction (X - longitudinal direction, Y - transversal direction).

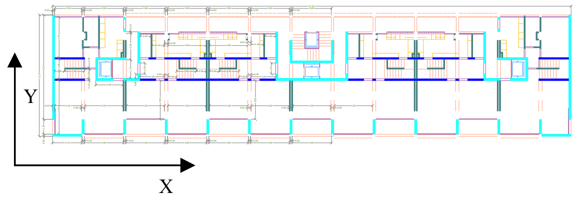


Fig. 7. Structural system (plan)

The twelve transversal plane frames have the same geometric characteristics for all beams and columns. However, three different frame-types (labelled A, B and C) were identified, according to reinforcement detailing differences.

A peculiar structural characteristic of the type of buildings, with direct influence in the global structural behaviour, is the ground storey without infill masonry walls. Furthermore, at the ground storey the columns are 5.5m height. All the upper storeys have an inter-storey height of 3.0m.

In the two structural models (X and Y) a concrete slab 1.25m width and 0.20m thick. A detailed definition of the existing infill panels were considered in the structural models.

For the building analysis in the transversal direction (Y), it was assumed an equivalent model defined as the association of the three frame-types, interconnected by rigid strut bars, as showed in figure 8. In this global model, the geometric and mechanical characteristics of each frame are multiplied by the number of occurrences of each frame-type.

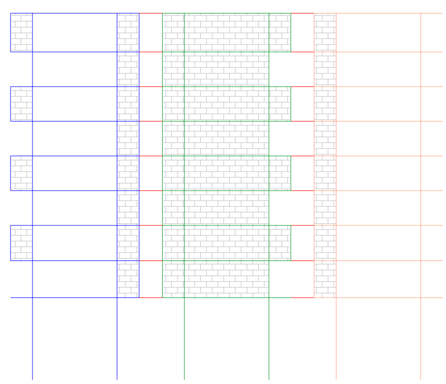


Fig. 8. Equivalent structural system for transversal direction (Y)

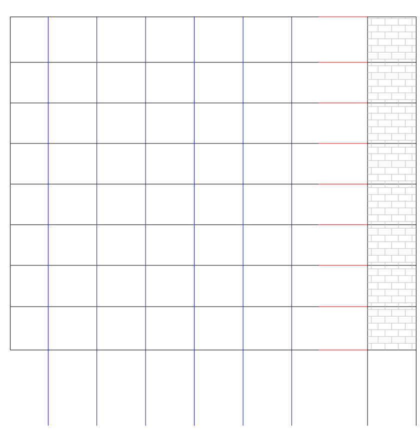


Fig. 9. Equivalent structural system for longitudinal direction (X)

For the analysis in the longitudinal direction (X), and because the double symmetry in plan, it was studied just one quart of the building. For the global model results a six columns structure linked at all storey levels by the RC slabs. No full-bay infill panels exist in the longitudinal direction. Therefore, an external simplified global infill masonry model was considered, as represented in figure 9, connected throw rigid struts to the RC structure.

STATIC LOADS, MASSES AND DAMPING

For the numerical analyses, constant vertical loads distributed on beams were considered in order to simulate the dead load of the self-weight including RC elements, and infill walls, finishing, and the correspondent quasi-permanent value of the live loads, totalising a value of 8.0kN/m^2.

The mass of the structure was assumed concentrated at storey levels. Each storey has a mass, including the self-weight of the structure, infill walls and finishings, and the quasi-permanent value of the live loads, of about 4Mtons. For the dynamic analysis, the storey mass is assumed to be uniformly distributed on the floors.

A viscous damping ratio of 1% was considered in the numerical analysis for each vibration mode. This value is smaller than those normally used in the linear dynamic analyses [Varum, 2003].

For each structural model, a Rayleight damping matrix, with 1% damping ratio for the first two natural modes, was considered, according with

$$[C] = \beta[K] + \alpha[M]$$

where the coefficients β and α are calculated such that 1% damping ratio in the first two modes of vibration is achieved. [K] and [M] are the stiffness and mass matrices of the structure, respectively.

NATURAL FREQUENCIES AND MODES

A first validation of structural numerical models can be achieved comparing the experimentally measured and the analytically estimated natural frequencies.

In table 1 are listed the four first natural frequencies computed, for each building direction.

To validate the numerical building models, in the two independent directions, it were measured the first natural structural frequency, with a seismograph and the ambient vibration. The measured first frequency is indicated in brackets in table 1.

TABLE 1: Natural frequencies for directions X and Y

Frequencies	Direction	
	Longitudinal X (Hz)	Transversal Y (Hz)
1st	1.08 (1.17)	1.75 (1.56)
2nd	5.67	6.41
3rd	6.32	8.14
4th	8.10	8.80

A good agreement was found between the experimentally measured frequencies (1.17Hz for longitudinal direction and 1.56Hz for transversal direction [Miranda et al., 2005]) and the frequencies estimated with the structural numerical models (1.08Hz for longitudinal direction and 1.75Hz for transversal direction), which constitutes the first validation of the numerical model. In figure 10 are represented the first natural mode for each direction.

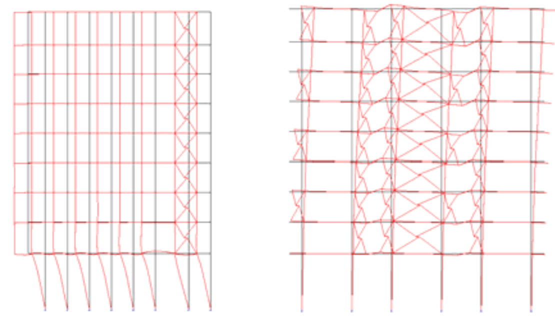


Fig. 10. Natural vibration modes ($f_{1,X}$ = 1.08Hz and $f_{1,Y}$ = 1.75Hz)

From the analysis of the first vibration shape modes, in both directions, it is clear that the seismic structural response will induce soft-storey mechanism behaviour. This conclusion will be confirmed with the earthquake analysis results in the next sections.

EARTHQUAKE INPUT SIGNALS

A family of earthquake input motions artificially generated for the experimental project ICONS [Varum, 2003] were adopted in this study. The ICONS accelerograms were artificially generated for medium/high seismic risk scenario in Europe [Carvalho et al., 1999]. In table 2 are listed the peak acceleration for each earthquake labelled with the corresponding return period. In figure 11 is illustrated the ICONS earthquake corresponding to the return period of 975 years.

TABLE 2: Peak acceleration of the ICONS earthquakes

Return Period (years)	Peak acceleration (g)
73	0.091
100	0.108
170	0.143
300	0.183
475	0.222
975	0.294
2000	0.380
3000	0.435
5000	0.514

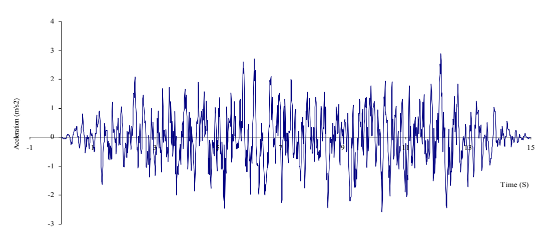
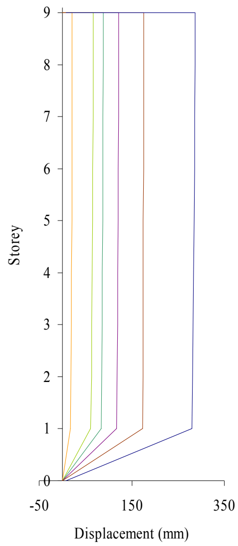


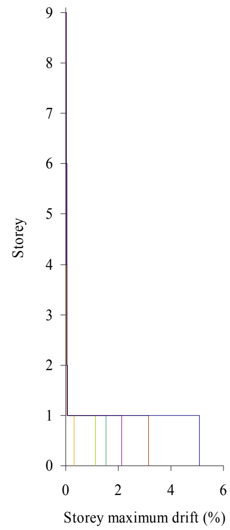
Fig. 11. Accelerogram - 975 years return period (ICONS series)

RESULTS ANALYSIS

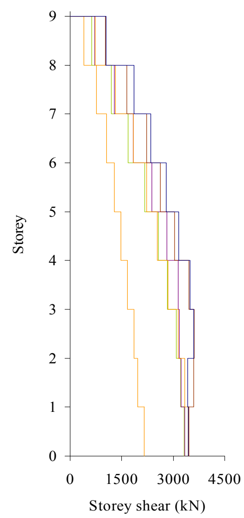
As observed previously, in the vibration shape modes analysis, the structural response of the building, in both directions, induces soft-storey mechanism behaviour. This leads to larger inter-storey drifts at the first storey and the upper storeys remain with very low deformation levels. Next, are presented the numerical results of the analysis, for the longitudinal (figure 12) and transversal (figure 13) directions, in terms of envelop deformed shape, maximum inter-storey drift, and maximum storey shear, for each earthquake input motion (73, 475, 975, 2000, 3000, 5000 years return periods).



a) envelop deformed shape

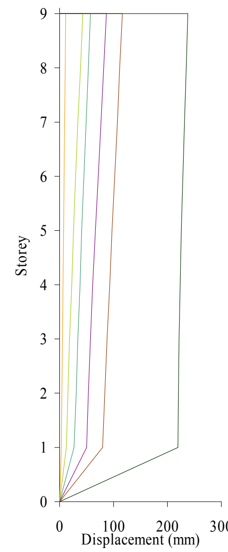


b) maximum inter-storey drift profile

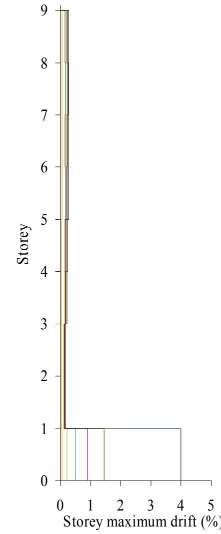


c) maximum storey shear profile

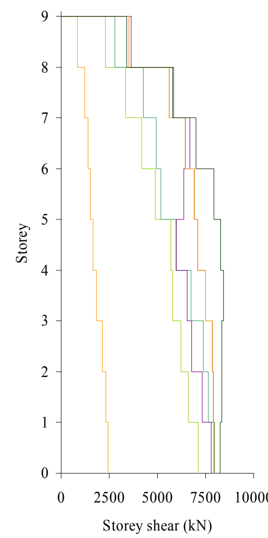
Fig. 12. Results for the longitudinal direction



a) envelop deformed shape



b) maximum inter-storey drift profile



c) maximum storey shear profile

Fig. 13. Results for the transversal direction

From the envelop deformed shape of the building and from the envelop of the inter-storey drift profile, for both directions, it can be observed that the deformation exigencies are concentrated at the first storey. In fact, the absence of infill masonry walls at the ground storey and the storey height, that is significantly larger than the upper storeys, induces an important geometrical structure irregularity.

For all the structural elements (columns and beams), the maximum shear force assumes a value inferior to the corresponding shear capacity, which confirms the safety of the columns in shear

Comparison between transversal and longitudinal analysis

In figure 14 are compared the vulnerability curves, for the longitudinal and transversal directions, for the maximum 1st storey drift, obtained from the numerical analysis

The results show that, for the 1st storey, the maximum inter-storey drift is always larger for the longitudinal direction, than for the transversal one. The building is more flexible in the longitudinal direction. The critical direction of the building is the longitudinal, because for a certain earthquake input level, the maximum deformations are always verified in the longitudinal direction.

In figures 15 and 16 are represented the vulnerability curves in terms of maximum 1st storey shear force and top displacement.

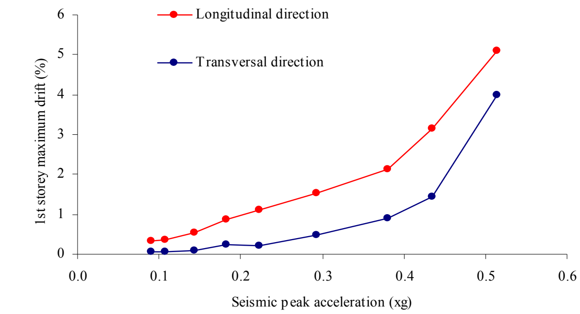


Fig. 14. 1st storey drift vs. peak earthquake acceleration

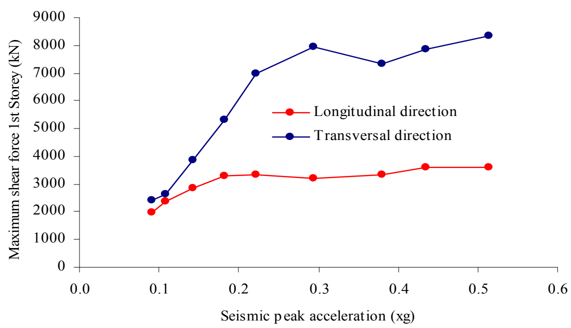


Fig. 15. 1st storey shear vs. seismic peak acceleration

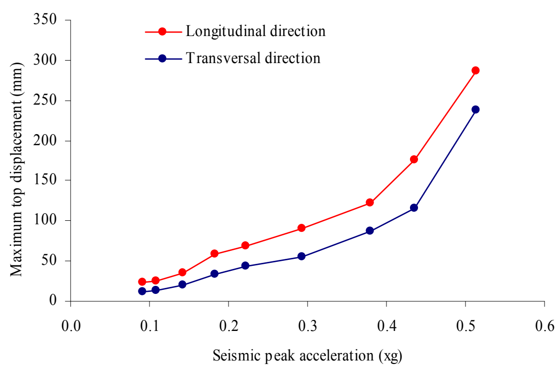


Fig. 16. Top displacement vs. seismic peak acceleration

Seismic safety assessment of the building

As presented in the previous section, for each direction, the building structure was analysed to a series of earthquakes with increasing intensities, in order to estimate damage levels.

The obtained results allow verifying the safety according to the hazard levels specified in VISION-2000 [SEAOC, 1995] and ATC-40 [1996] documents.

In tables 3 and 4 are presented the acceptable drift limits for each structural performance level according to the ATC-40 and in VISION 2000, respectively.

TABLE 3: Drift limits according to the ATC-40 [1996]

	Performance Level			
	Immediate Occupancy	Damage Control	Life Safety	Structural Stability
Drift Limit	1%	1-2%	2%	$0.33\frac{V_i}{P_i} \approx 7\%$

TABLE 4: Drift limits according to the VISION-2000 [SEAOC, 1995]

	Performance Level			
	Fully Operational	Operational	Life Safe	Near Collapse
Drift Limit	0.2%	0.5%	1.5%	2.5%

The VISION-2000 [SEAOC, 1995] document presents a performance objectives matrix for buildings, definig three levels of performance objectives (basic, essential hazardous, and safety critical). For the building under study the safety was investigated just for the Basic Performance Objectives, proposed at the VISION-2000 (table 5), which are marked with a "X".

TABLE 5: Basic Performance Objectives for buildings according to VISION-2000 [SEAOC, 1995]

		Fully Operational	Operational	Life Safe	Near Collapse
Earthquake design level	Frequent (43–yrp)				
	Occasional (72-yrp)		X		
	Rare (475-yrp)			X	
	Very rare (970-2000 yrp)				X

In figure 17 are represented the vulnerability functions in terms of 1st storey maximum drift, already presented in figure 14, with indication of the safety limits proposed at the ATC-40 and VISION-2000 recommendations (indicated in tables 3 and 4).

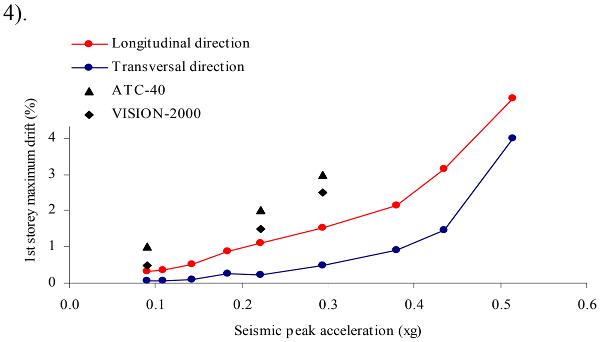


Fig. 17. 1st storey drift vs. peak earthquake acceleration and safety limits

Comparing the maximum storey drift estimated with the safety limits proposed at the ATC-40 and VISION-2000 recommendations (figure 17), it can be concluded that the building safety is guaranteed in both directions.

Concluding Remarks

The global structural safety of a modern architecture building at the Infante Santo Avenue was investigated.

Although the results indicate the building safety for the Basic Objectives according to the international seismic recommendations (ATC-40 and VISION-2000), it should be pointed out that additional analyses have to be performed.

The input motion earthquakes adopted for these analyses can be not fully representative of the possible seismic action in Lisbon. In other way, the level of structural damage does not dependent just of the peak ground acceleration of the earthquake. Additional analyses should be performed using other earthquake motions.

Shear capacity was verified for all the input motions. However, the model adopted for these analyses does not consider the geometric non-linearity, which can increase significantly the moments in columns and global storey lateral deformations (drifts). Therefore, to guarantee the seismic safety verification of the building, it is judged focal to verify the results using a model that considers the geometrical non-linearities.

References

Tostões, A., Os Verdes Anos na Arquitectura Portuguesa dos Anos 50, Porto, FAUP, 1997 (in Portuguese)

Tostões, A. (ed.), Arquitectura Moderna Portuguesa, Lisboa, IPPAR, 2004 (in Portuguese)

Le Corbusier, Vers Une Architecture, Paris, Flammarion, 1923

Varum, H., Modelo numérico para análise de pórticos planos de betão armado, MSc Thesis, Civil Engineering Department, University of Porto, 1996 (in Portuguese)

Costa, A.G., Análise Sísmica de Estruturas Irregulares, PhD Thesis, Civil Engineering Department, University of Porto, 1989 (in Portuguese)

Miranda, L., Rodrigues, H., Fonseca, J. Costa, A., Relatório de Inspecção ao Bloco 3 do complexo do Infante Santo, 2005 (in Portuguese)

Rodrigues, H.; Varum, H.; Costa, A., Numeric Model to account for the influence of infill masonry in the RC Structures Behaviour, Congreso Métodos Numéricos en Ingeniería, 2005

Zarnic, R.; Gostic, S., Non-linear modelling of masonry infilled frames, 11th ECEE; Paris, France, ISBN 90 5410 982 3, A.A. Balkema, Rotterdam, 1998

Varum, H., Seismic Assessment, Strengthening and Repair of Existing Buildings, PhD Thesis, Department of Civil Engineering, University of Aveiro, 2003

Infante Santo residential building (Block 3), Drawings and Descriptive and Justificative Memoir, 1950-1956

Carvalho, A., Campos-Costa, A., Oliveira, C.S., A stochastic finite–fault modelling for the 1755 Lisbon earthquake, 13th World Conference on Earthquake Engineering Vancouver, B.C., Canada, 2004

Carvalho, E.C.; Coelho, E.; Campos-Costa, A., Preparation of the full-scale tests on reinforced concrete frames - Characteristics of the test specimens, materials and testing conditions. ICONS report, Innovative Seismic Design Concepts for New and Existing Structures, European TMR Network - LNEC, Lisbon, 1999

SEAOC, Performance based seismic engineering of buildings, Part 2: Conceptual framework - Vision 2000 Committee, Structural Engineers Association of California, Sacramento, California, 1995

ATC-40, Seismic evaluation and retrofit of concrete buildings - Applied Technical Council, California Seismic Safety Commission, Report No. SSC 96-01 (two volumes), Redwood City, California, US, 1996